29 CONNECTION DESIGN – DESIGN REQUIREMENTS

1.0 INTRODUCTION

Steel sections are manufactured and shipped to some standard lengths, as governed by rolling, transportation and handling restrictions. However, most of the steel structural members used in structures have to span great lengths and enclose large three-dimensional spaces. Hence connections are necessary to synthesize such spatial structures from one- and two-dimensional elements and also to bring about stability of structures under different loads. Thus, connections are essential to create an integral steel structure using discrete linear and two-dimensional (plate) elements.

A structure is only as strong as its weakest link. Unless properly designed, the connections joining the members may be weaker than the members being joined. However, it is desirable to avoid connection failure before member failure for the following reasons:

- To achieve an economical design, usually it is important that the connections develop the full strength of the members.
- Usually connection failure is not as ductile as that of steel member failure. Hence it is desirable to avoid connection failure before the member failure.

Therefore, design of connections is an integral and important part of design of steel structures. They are also critical components of steel structures, since

- They have the potential for greater variability in behaviour and strength,
- They are more complex to design than members, and
- They are usually the most vulnerable components, failure of which may lead to the failure of the whole structure.

Thus designing for adequacy in strength, stiffness and ductility of connections will ensure deflection control during service load and larger deflection and ductile failure under over-load. Hence, a good understanding of the behaviour and design of joints and connections in steel structures is an important pre-requisite for any good design engineer. This chapter gives an overview of the design of connections in steel structures. The following five chapters deal with bolted and welded connections in greater detail.

2.0 COMPLEXITIES OF STEEL CONNECTIONS

Margins of safety of any design, in particular that of connection, involves uncertainty due to random nature of (a) the forces acting on the structure and (b) the actual strength of the joint designed.

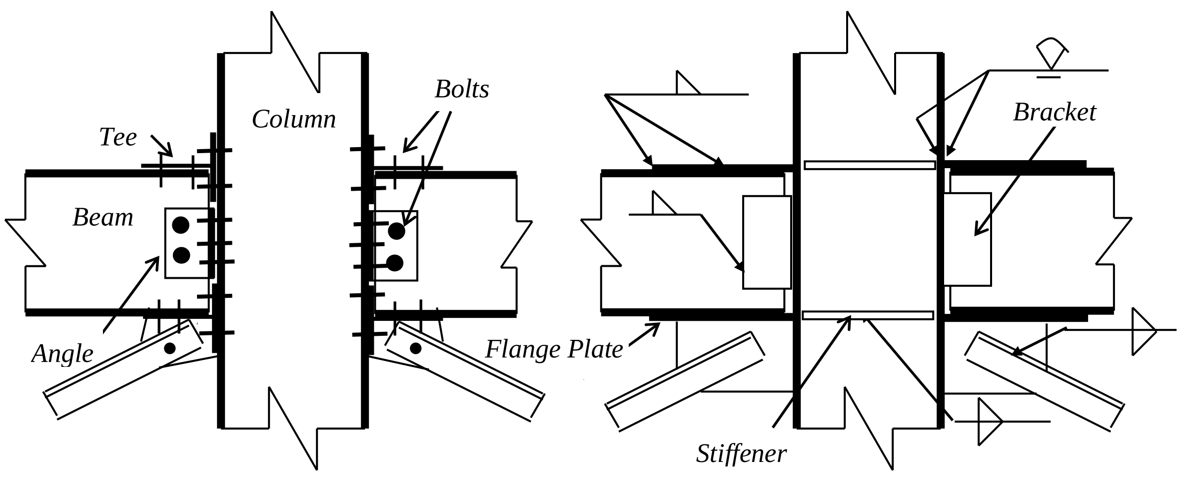
The randomness of the loads has been discussed in an earlier chapter; that of the actual strength is due to the variability of the dimensions of the elements and that of the strength of constituent material as well as errors due to simplification in analysis and design.

The reasons for the high uncertainty and complexity of the connection are:

- Complexity of connection geometry
- Geometric imperfections
- Residual stresses and strains

2.1 Complexity of connection geometry

The geometry of connections is usually more complex than that of the members being joined (Fig.1). The stress analysis of the joint is complicated by the (locally) highly indeterminate nature of the joint, non-linear nature of the behaviour due to lack of fit, local yielding etc. and stress concentration due to discontinuity in elements around bolt holes and weld profiles.



(a) Bolted Connection *(b) Welded Connection*

Fig. 1 Complex Beam to Column Connections

2.2 Geometric imperfections

The following factors contribute to the geometric imperfections in connection:

- Bow in the beam or column as rolled
- Lack of fit in black bolts in clearance holes
- Gaps in the connecting plate and the surface of the member to be connected to, due to fabrication errors, welding distortions, and tolerances allowed for ease of fabrication and erection

2.3 Residual Stresses and Strains

Residual stresses and strains are inherent features of steel joints due to differential cooling after the hot rolling, gas cutting and welding stages. The residual stresses cause premature local yielding and the residual strains cause distortions and lack of fit.

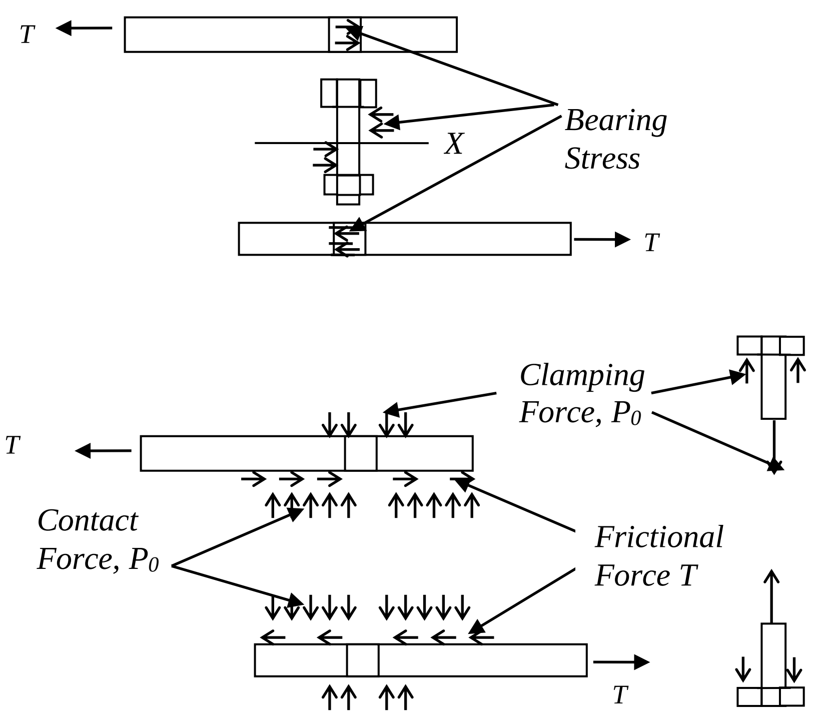
3.0 TYPES OF CONNECTIONS

Connections are normally made either by bolting or welding. Bolting is common in field connections, since it is simple and economical to make. Bolting is also regarded as being more appropriate in field connections from considerations of safety. However, welded connections, which are easier to make and are more efficient, are usually resorted to in shop fabrications.

3.1 Bolted Connections

Two types of bolts are used in bolted connection. The most common type is bearing bolts in clearance holes, often referred to as ordinary bolts or black bolts. They are popular since they are economical, both in terms of material and installation costs.

The force transfer mechanism under shear is as shown in Fig. 2(*a*). The force is transferred by bearing between the plate and bolts at the bolt holes. The bolts experience single or double shear depending upon the plate configuration. The failure may be either by shearing of the bolts or bearing of the plate and the bolt.



(a) Bearing Connection

(b) Friction Connection

Fig. 2 Bolt Shear Transfer Mechanism

The main disadvantage of bearing type of bolted connections is that the elements undergo some slip even under a small shear, before being able to transfer force by bearing. This is due to clearance between the bolts and the holes. Such a slip causes increased flexibility in the lower ranges of load and unexpected joint behaviour in some situations. In such cases high strength friction grip (HSFG) bolts are used.

In HSFG bolted joints, high strength bolts (8G or 10K grade) are pre-tensioned against the plates to be bolted together, so that contact pressure is developed between the plates being joined [Fig. 2(*b*)]. When external shear force is applied, the frictional resistance to slip between the plates prevents their relative slip. These bolted joints achieve higher stiffness in shear because of frictional resistance between the contact surfaces. Only when the externally applied force exceeds the frictional resistance between the plates, the plates slip and the bolts bear against the bolt holes. Thus even after slip, there is a reserve strength due to bearing.

The HSFG bolts are expensive both from material and installation points of view. They require skilled labour and effective supervision. Due to their efficient force transfer mechanism they have become very popular recently. Moreover, their performance is superior under cyclic loading compared to other forms of jointing. This is discussed later.

3.2 Welded Connections

Welded connections are direct and efficient means of transferring forces from one member to the adjacent member. Welded connections are generally made by melting base metal from parts to be joined with weld metal, which upon cooling form the connection. The welded connections in a majority of the cases may be categorised as fillet weld or butt (or groove) welds as shown in Fig. 3.

Fillet welds, as shown in Fig. 3(*a*), are made against two surfaces of adjacent plates to join them together. The merits of the fillet welds are:

- no prior edge preparation is necessary,
- simple, fast and economical to make, and
- does not require very skilled labour.

The demerits of fillet welds are:

- not appropriate to transfer forces large in magnitude,
- poorer performance under fatigue loading, and
- less attractive in appearance.

Butt welds, as shown in Fig. 3(*b*), are made by butting plate surfaces against one another and filling the gap between contact surfaces with weld metal, in the process fusing the base metal also together. In order to ensure full penetration of the weld metal, normally the contact surfaces are cambered to obtain gap for the weld metal to flow easily.

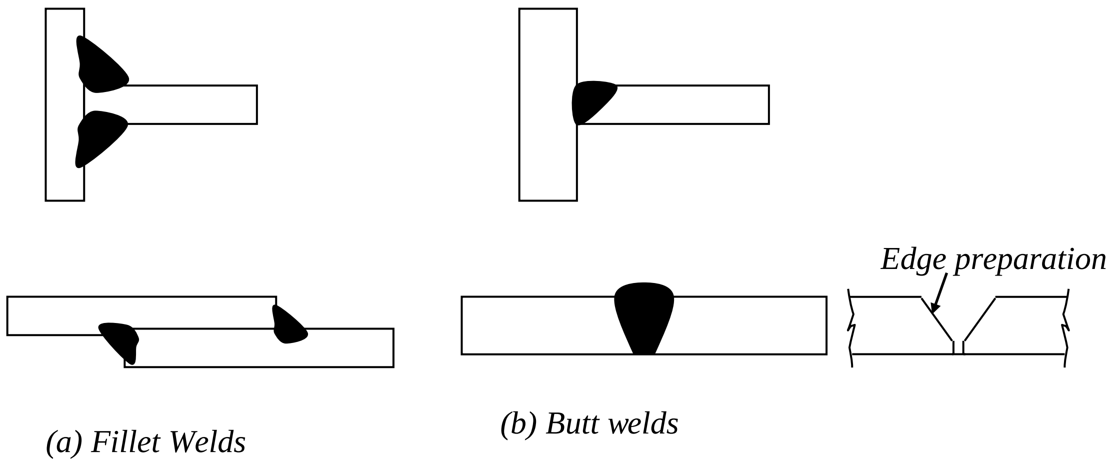


Fig. 3 Typical welded Connections

The merits of butt welds are:

- easily designed and fabricated to be as strong as the member,
- better fatigue characteristics, compared to fillet welds,
- better appearance, compared to fillet welds, and
- easy to detail and the length of the connection is considerably reduced.

The demerits of the butt welds are:

- more expensive than fillet welds because of the edge preparation required, and
- require more skilled manpower, than that required for filled welds.

3.3 Riveted Joints

Riveted joints are very rare in modern steel construction practice. The behaviour and design of riveted connections are very similar to bearing type of bolted constructions. Since structural rivets are driven hot, the rivet shank expands to fill the hole while being driven. Hence, while calculating rivet strength, the hole diameter and not the nominal rivet diameter is used. Due to this, the slip in riveted joints is less than in bearing type of bolted joint. Further, in the process of cooling, the rivet shank length reduces, thereby causing some clamping force, as in HSFG.

Riveting has been traditionally limited to railway bridges in India. However, with the introduction of HSFG bolts, which are better suited under cyclic loading than rivets, their use is discontinued even in railway bridges in most countries.

3.4 Moment Resisting Connections

Moment resisting connections between beams and columns in multistoried buildings are very common. These connections may be made using bolting or welding. Depending upon the type of joining method and elements used to make the joint, the flexibility of the joint may vary from *hinged* to *rigid joint* condition. The moment at the joint, M, may vary between rigid joint moment, M_r [Fig. 4(a)], and zero value [Fig. 4(b)] and the relative rotation between members at the joint, θ, may vary between zero [Fig. 4(a)] and hinged joint rotation, θ_h [Fig. 4(b)].

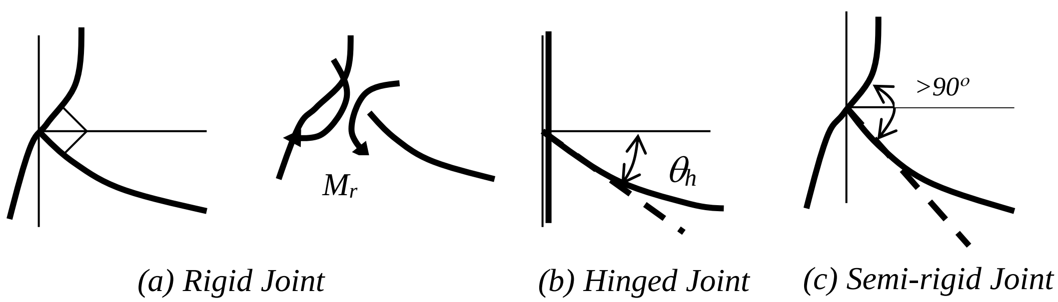


Fig. 4 Types of Beam to Column Joints

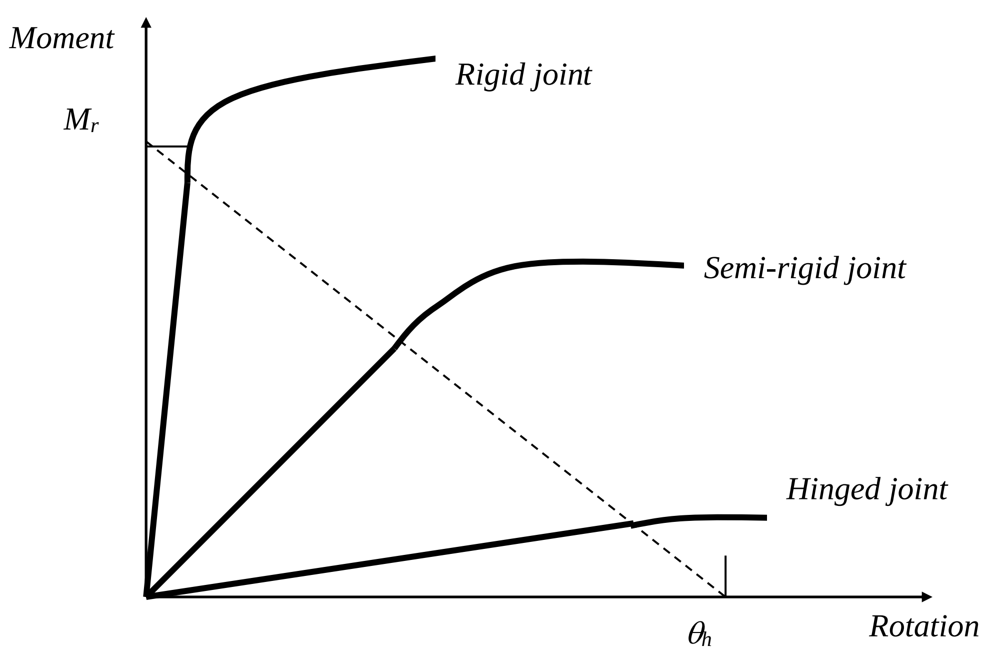


Fig. 5 Moment Versus Joint Rotation

In practice the joints are neither ideally hinged nor ideally rigid. In fact all the joints exhibit some relative rotation between members being joined [Fig. 4(*c*)]. This is due to the deformation of elements in the joint. The moment versus relative joint rotation of different types of connections is shown in Fig. 5. Any joint developing more than 90 % of the ideal rigid joint moment is classified as rigid and similarly any joint exhibiting less than 10 % of the ideal rigid joint moment is classified as hinged joint; and the joint developing moments and rotations in between are referred as semi-rigid. Based on test results and theoretical studies, moment rotation relationship for different standard connections exhibiting semi-rigid behaviour has been presented in literature.

4.0 CONNECTION DESIGN PHILOSOPHIES

Traditional methods of analysis of connection stresses were based on the following assumptions:

- Connected parts are rigid compared to connectors themselves and hence their deformations may be ignored
- Connectors behave in a linear-elastic manner until failure.
- Connectors have unlimited ductility.

However, in reality, connected parts such as end plates, angles etc. are flexible and deform even at low load levels. Further, their behaviour is highly non-linear due to slip, lack of fit, material non-linearity and residual stresses. Ductility of welds in some orientation with respect to direction of loads may be very limited, (eg. Transverse fillet welds)

Eventhough truss joints are assumed to be hinged the detailing using gusset plates and multiple fastener and welding does not represent hinged condition. However, in practice the secondary moment associated with such a rigid joint is disregarded unless the loading is cyclic.

The complexity and variability in strength of connections require a rational design philosophy to account for their behaviour. Keeping in view the large number of joints to be normally designed in a structure and the considerable variability in the design strength, any sophisticated analysis is neither desirable nor warranted. The design should ensure that equilibrium is satisfied, slenderness of the elements is consistent with the ductility demand and the deleterious effects of stress concentration on fatigue strength is considered in cyclically loaded structures. The following approach is consistent with connection design requirements in most general cases encountered in practice in statically loaded systems.

The steps to be followed in the proposed rational design approach are enumerated initially. These are illustrated using a simple framing angle connection between a beam and a column of a framed building designed to transfer a shear force of V, as shown in Fig. 6.

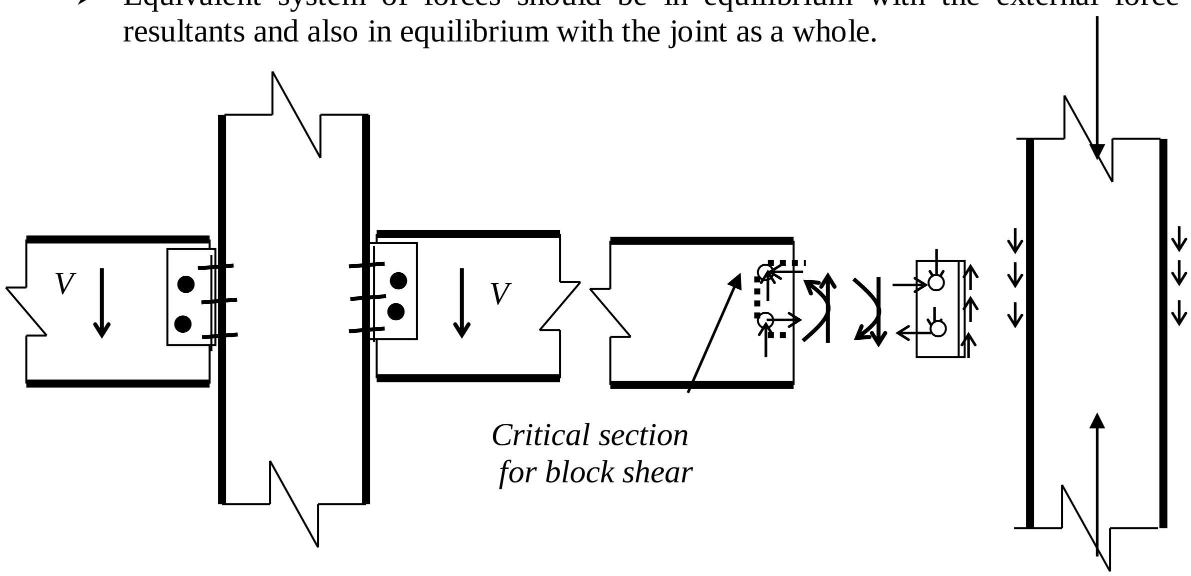
4.1 Steps in Transfer of Member Forces to Joints

Overall connection behaviour should be clearly understood in order to effectively and efficiently design connections following simple procedure, such as the one discussed below. To start with, the stress resultants (moment, shear, torsion, axial force etc.) transmitted by the members to be joined are to be determined. Normally analysis for forces is carried out using a model wherein members are represented by their centroidal line. Thus the calculated forces in the joints are at the intersection of centroidal line of members meeting at the joint. Therefore the effect of the size of the joint in reducing the design forces to correspond to that at the face of the joints, if substantial, has to be considered. The force resultants thus obtained should be replaced by an equivalent system of forces on the elements of the joint. In carrying out this replacement by an equivalent system of forces in the joint elements, the following are to be considered.

- The distribution of forces in the elements being connected is considered first. (For example, in the case of a beam, major proportion of the bending moment is carried by the flanges and the major proportion of shear force is carried by the web. Hence, the equivalent forces may be assumed to act on the corresponding elements at the interface)

- The equivalent system of forces should be consistent with the flexibility of the joint. For example plate elements are stiffer in resisting forces acting in their plane than in resisting forces normal to the plane. Hence most of the forces acting at a junction would be transferred to the plate in the plane of the force and little is transferred to a plate perpendicular to the force.

- Equivalent system of forces should be in equilibrium with the external force resultants and also in equilibrium with the joint as a whole.



(a) Connection (b) Freebody Diagram

Fig. 6 Simple Framed Angle Shear Connection

In the framing angle joint shown in Fig. 6, the shear from the beam web acts eccentric with respect to reaction from the column flange causes couple. The framing angle leg connected to the column is weak in resisting any moment normal to the plane of the leg. Hence the moment at this flange connection may be assumed be negligible and only the shear force, *V*, may be assumed to be acting on the leg connected to the column flange. In the framing angle connection with the web of the beam, the forces act in the plane of the framing angle and in the plane of web of the beam. Hence both shear and the moment to equilibrate the couple due to eccentricity of shear in the framing angle can be resisted by this connection.

4.2 Determination of Force flow in the joint

Once the equivalent forces in the interface elements are obtained, the flow path of the forces through the elements in the joint is to be established by using equilibrium and simplifying assumptions regarding the force sharing, based on their relative stiffness as discussed earlier. At each stage, each element in the force flow path should be checked to ensure that they have

(a) adequate strength to withstand the force and
(b) adequate ductility to redistribute the forces to parallel elements in case of overload.

The strength and ductility evaluation is to be done for all component plates in the force path as well as all the joining elements such as bolts and welds.

As mentioned earlier the distribution of forces to different elements in the joints is complex due to highly indeterminate interaction of different element. Hence in practical joint design, the force flow analysis is based on simplifying assumptions with regard to sharing of forces. These assumptions may be at variance with the actual stresses in the elastic range. Hence it is important that adequate ductility is exhibited by all elements to redistribute the forces among alternate elements in case of over-load. This step in the framing angle joint example in Fig. 6(*a*) is illustrated in Fig. 6(*b*), in the form of free body diagram of all the elements and the force flow in the elements, while satisfying equilibrium.

Using these free body diagrams, the stresses/forces in the elements in the joint can be evaluated and compared with their respective strength, as given below:

- The bolts are assumed to share the shear force equally. Due to misfit and clearance between the bolts and the holes, in the elastic range, this need not be true. However, as long as the bolts behave in a ductile fashion, the assumption of equal sharing of shear by bolts is valid, before failure, due to plastification.

- The framing angle experiences shear and bending due to the eccentricity of the shear load. The section with holes corresponding to the bolts connecting framing angle to the beam web is the critical section, since this section experiences shear and moment. The "Strength of Materials" approach to calculate shear and bending stresses is not strictly appropriate here due to the deep beam nature of the bending behaviour of angle leg. Nevertheless, usually stresses in the framing angle are calculated based on "Strength of the Materials" concepts, due to very small value of these stresses. These stresses are usually very nominal and hence frequently need not be checked.

Bolts connecting framing angle with the beam web are subjected to the same shear force and moment in the angle legs. This is an eccentric bolted connection. The vertical shear and horizontal shear in the bolts due to the shear force and moment, respectively, are calculated and the resultant shear in the bolt is evaluated. This again is based on the rigid angle and flexible bolt assumption and the method of superposition. The maximum resultant shear force in the bolt has to be checked against the shear strength of the bolt.

The stresses in beam web and column flange can be checked at the location of bolt force transfer, by following block shear method at critical sections as shown in Fig. 6(*b*). Usually these stresses would be very nominal.

5.0 BEHAVIOUR OF ELEMENTS IN CONNECTIONS

Many local elements such as end plates, framing angles, stiffeners are used in a connection design. These elements on the load path have to perform the function of transferring forces imposed upon them. Frequently forces are distributed somewhat arbitrarily between parallel elements in the load path. In order to redistribute the loads as assumed and in order to avoid sudden failure, these elements have to behave in a ductile fashion in case of overloading.

5.1 Distribution of Forces in Elements

The joints are locally complex and theoretically exact calculation of element force/stress is a highly indeterminate analysis problem, making exact analysis of a joint impractical in day-to-day design. Theoretically exact analysis methods and experimental studies are used for research to develop a better understanding of the force flow and simplified connection design procedures. One often makes simplifying assumptions consistent with the internal behaviour of the elements and relies heavily on ductility to redistribute over-load on any element. This process requires a good understanding of the following:

- Free body diagram and equilibrium analysis of elements in the load transfer path,
- Relative stiffness of elements in the load transfer path, and
- Ductility demand on the elements and the consequent slenderness limitation.

The simplified analysis steps are illustrated through a few examples. Let us consider an interior beam to column moment resisting connection of a frame, as shown in Fig. 7. It is seen that shear and bending moment should be transferred from the beams to the column as shown. We know that a major portion of the bending moment in a beam is transferred through bending stresses in flanges and a major portion of the shear force in the beam is transferred through shear stress in the web, as shown. Equal and opposite forces act on the column flanges, as shown in Fig. 7.

The concentrated beam flange forces (*C* and *T*) have to be transferred as shear to the column web, since the column web plate is the stiff element in that plane in the load path. The transfer to the column web is through column flanges, which may cause excessive bending of column flanges and excessive bearing in the column web flange junction. In order to overcome this, we often use stiffener plates, S_1 and S_2 as shown.

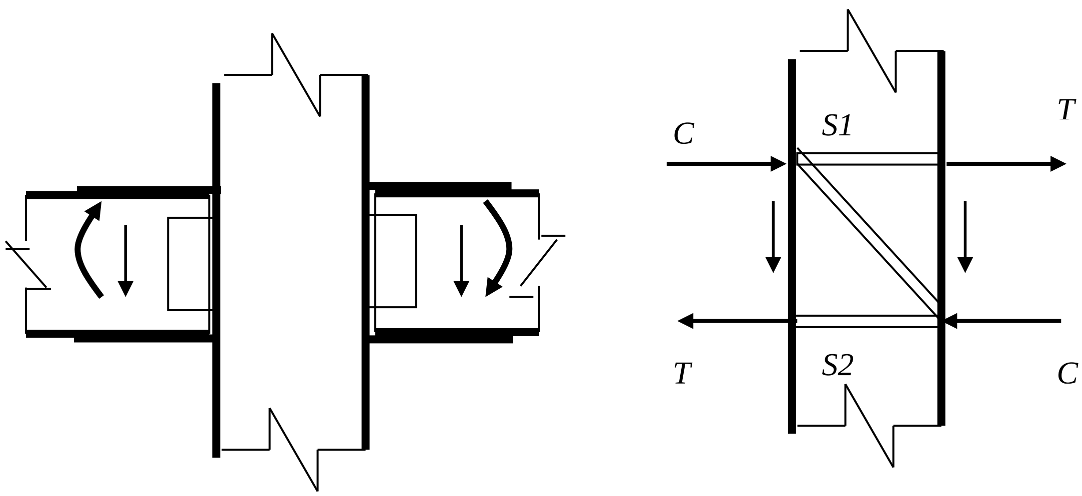


Fig. 7 Elements in Connections

The forces *T* and *C* may be either assumed to be fully transferred by the stiffeners provided or the balance force in excess of the bearing capacity of the web and bending capacity of the flange may be assumed as the design force in the stiffeners. The assumption made dictates the ductility requirement of the stiffener. If the entire force is assumed to be transferred by the stiffener, the actual force in the stiffener in the elastic range will be less than this and hence only semi-compact design requirement with regard to the b/t ratio has to be satisfied by the stiffener (see chapter on plate buckling), since it needs only to carry the load without local buckling.

If, however, the stiffener is designed for forces in excess of the capacity of the flange and web of the column, the design force on the stiffener is usually an underestimation of the actual force experienced by it in the elastic range. This is due to the higher rigidity of the stiffener compared to the column flanges. Consequently, the stiffener should deform plastically on over-loading so that the load on stiffener, in excess of what it has been designed for, can be redistributed. Hence stiffeners should not only sustain the force but also plastically deform (adequate ductility is needed) in order to redistribute the force and hence the slenderness of the stiffener should meet the compact plate element requirement (see the chapter on plate buckling).

The unbalanced beam moment transferred by the beam to the column at the junction causes shear (V = C+T in Fig. 7), locally at the joint in the column web. This may be in excess of shear capacity of the column web. Hence the column web may have to be locally thickened or provided with a diagonal stiffener, as shown in Fig. 7. Further, the welds between the stiffeners and the column flange should be sufficiently large so that they remain elastic during the plastic deformation of the stiffener, discussed earlier.

The shear from the beam is directly transferred to column B through column flanges, as an additional axial compression. Thus, all the elements in the force transfer path across the joint should be ensured to have adequate strength, stiffness and ductility, to perform the function based on rational simplifying assumptions.

6.0 COST OF CONNECTIONS

Usually cost of fabrication and erection constitute as high as 50% of the total cost of steel structures, per tonne of material used. Hence, designers of connections have a great responsibility in reducing the overall cost of steel structures.

Factors affecting design cost:

Important factors affecting connection design costs are discussed below:

- Connection design takes up a significant part of the overall design cost of steel structures and decisions made at this stage considerably influence the fabrication and erection costs.
- The connection designs should be done using simple and standard cases, so that using design tables, connections can be designed and detailed rapidly. Such tables considerably reduce repetitive calculations, improve accuracy and speedup fabrication.

Factors affecting fabrication/erection costs:

Important factors in improving productivity, decreasing cost of fabrication and erection of connection work are discussed below:

- Repetitive use of standard detail.

The repetitive use of standard details spread the cost of learning, cost of setup, cost of templates etc. over a large number of products/components to be fabricated, thus reducing the cost and time required for fabrication. Special, complicated and precise fitting details should be avoided or minimised.

- Ease of joining

The detail should provide easy access to welding and bolting. The positioning of members should be simplified with temporary supports to facilitate quick release of the handling equipment, ease of adjustment and alignment and quick joining.

- Appropriate mix of automatic and manual fabrication

The productivity of numerically controlled automatic machineries (NC machines), and continuous submerged arc welding is very high compared to manual methods. The quality is usually superior. However, their setup costs are high. Hence, automatic fabrication methods are appropriate in large volume jobs. For example, a large number of framing angles can be cut and drilled to the same part detail using NC machines and long continuous fillet weld between plate girder web and flange can be done using an automatic submerged welding machine, economically. In the Indian market such machines are not widely available. Most fabrication shops still work with outdated equipment and require capital equipment infusement to bring about efficiency and economy in shop fabrication and erection.

Manual methods take less setup time and unit time costs are low, but productivity and quality are also low. Hence the manual methods are appropriate in fabricating a smaller number of elements or in shorter welds, such as web stiffener welding.

- Choice of connection method

Generally welded connections are more direct and more efficient, but require more elaborate preparation and machinery compared to bolted connection. This has generally led to the use of welding in shop and on ground field connections and the use of bolting at the erection connections.

There are exceptions to this general tendency. For example, if only a few angle trusses are to be fabricated, then pre-drilling of the members in shop, based on theoretical calculations of geometry of members and connection sizes and site assembly subsequently by bolting would be economical compared to laying the truss out and aligning the members appropriately and welding them together on ground. On the other hand, welded fabrication may be economical in the case of a large number of trusses fabricated to the same detail, wherein the higher cost incurred for templates, layout and welding are spread over the larger number of units to be fabricated.

- Choice of shop versus site fabrication

Shop fabrication is faster, cheaper, has better quality and higher productivity. In India, the cost advantage of shop fabrication is partly off set by differential excise duty rates between the shop and site fabricated components, as well as low productivity equipment and process used in shop practices.

Transportation cost also dictates the economy of shop fabrication. The transportation cost is governed by distance to be transported, weight and volume of component to be transported. Instead of transporting a very long girder from a shop, it can be shop fabricated in shorter segments and joined at field using bolting or welding, to achieve greater economy. Fittings such as framing angles can be pre-attached to one of the members being joined (say the web of beam) at shop using welding and connected at field to the other member by bolting.

- Other Factors

Difficult connection details cause difficulty in understanding and execution at site. This may lead to frustration, carelessness, poor quality connections, and also mistakes leading to delay, cost of repair and failure. Prefabricated units to be connected at site should be of nearly uniform weight so that handling capacity of the cranes is fully utilized, improving the productivity of the handling equipment available.

HSFG bolted connections involve higher material cost, more skilled labour, more complex equipment, higher level of inspection, when compared to ordinary bolts. Hence its use should be restricted to special situations such as high forces and fatigue environment. Otherwise, at site, black bolts in clearance holes are preferred. Usually, the same grade of bolt and only a few standard sizes should be used at site, in order to reduce complexity of erection, maintenance of inventory of different size bolts and mistakes in connection.

7.0 SUMMARY

Sound connection design is essential for safety and economy of steel structures. Economical connection designs mostly take into account practicalities of fabrication and erection. True behaviour of connections is complex, variable and very difficult to analyse exactly. However, the connection design should be simple and straightforward, based on a clear understanding of the load transfer path, the effect of stiffness of elements in the path on the force distributed to the elements in the connection and the effect of ductility on the connection behaviour. The detailing of connection should be simple and be based on repetitive use of standard practices to facilitate ease of fabrication and erection, thus accure speed and economy to the project.

8.0 REFERENCES

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